An evaluation of pile load capacity estimates using CPTu and DMT methods in silty clay in High Prairie, Alberta

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ABSTRACT
Estimates of the ultimate axial pile load by direct and indirect methods are compared to the results of two static full scale load tests on CFA piles installed in soft to medium silty clay. The results show that both direct and indirect methods of design can provide reasonable estimates of the ultimate capacity of CFA piles in the clay soils in High Prairie, Alberta. Of the indirect methods, the α-method with undrained strength obtained from flat DMT data and an expression for α developed for CFA piles gave estimates of the failure loads to within ±5%. The β-method gave appropriately conservative predictions of capacity. Of the direct methods, the LCPC method also gave estimates of failure load within ±5% provided no limits were placed on the shaft friction.

RÉSUMÉ
Les évaluations de la charge axiale finale de pieux par des méthodes directes et indirectes sont comparées aux résultats de deux essais statiques a pleine échelle sur des pieux CFA installés dans de l'argile limoneuse molle à moyenne. Les résultats prouvent que les deux méthodes directes et indirectes de conception peuvent fournir des évaluations raisonnables de la capacité finale de pieux CFA dans les sols d'argile dans la High Prairie, Alberta. Des méthodes indirectes, la méthode-α avec la résistance non drainée obtenue à partir des données de DMT plat et une expression pour α développé pour des pieux CFA ont donné des évaluations des charges de rupture dans le ±5%. La méthode-β a donné des prévisions convenablement conservatrices de la capacité. Des méthodes directes, la méthode de LCPC a également donné des évaluations de charge de rupture dans le ±5% pourvu qu' aucune limite soit placée sur le frottement d'axe.

1 INTRODUCTION

This paper examines the applicability of common pile design methods for prediction of the axial capacity of Continuous Flight Auger (CFA) piles constructed in soft to firm silty clay in High Prairie, Alberta. Pile capacities are derived directly from the results of piezocone penetration tests (CPTu) and flat dilatometer tests (DMT) and indirectly using estimates of fundamental soil parameters. The estimates of axial pile capacity are compared to the results of full scale pile load tests to failure on two CFA piles.

The in situ tests used for the predictions were performed 1 month after completion of the pile load test program. Following the classification of predictions by Lambe (1973), the estimates of ultimate pile load discussed in this paper represent Type C1 predictions.

2 SOIL CONDITIONS

The site is located near the east end of the Town of High Prairie in northern Alberta. The Alberta Geological Survey (2008) indicates that the area is located above a buried bedrock valley about 10 km wide and 170 m deep. The infill of the bedrock channel consists mostly of clay but sand and gravel deposits were also found at the base of the bedrock channel. The surficial geology consists of fluvial sediments transported and deposited by streams and rivers in a generally flat to gently rolling topography. These sediments include stratified sand and gravel, silt, clay and organic sediments occurring in a channel and post glacial flood plains, terraces, fans and deltas.

Site investigation revealed soft to stiff, medium to high plastic clay interbedded with very loose to compact silty sand layers. The soft to stiff clay extended to about 16m and became stiffer and high plastic below that depth to the termination of the test hole (about 20 to 24 m). Very loose to compact sand layers ranging in thickness from 2.8m to 7.8m were found within the upper 13 m. The groundwater level was measured several days after the investigation to be at about 4m below ground surface.

3 PILE LOAD TEST PROGRAM

Two Continuous Flight Auger (CFA) test piles (TP1 and TP2) were installed to 14 m and 18 m depth, respectively. Six CFA reaction piles (RP1 to RP6) were installed to 18 m depth. The test layout is shown in Figure 1. A 0.5 m thick gravel pad was first placed on the test area to support the construction equipment. Pile tests to failure were conducted 14 days after construction. Axial load was applied by a hydraulic jack with load measured using a strain gage load cell. Pile head deflections were monitored using mechanical dial gauges. The ultimate axial compression load of the test piles determined by the Davisson method was 1.01 MN for TP1, and 1.35 MN for TP2. A more detailed description of the test procedure and discussion of the results are provided in a companion paper by Padros & Papanicolas (2008).

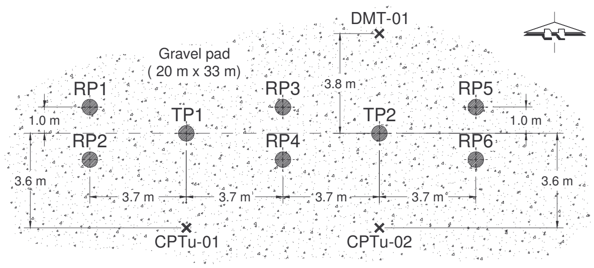

Figure 1. Test pile layout and location of CPTu and DMT soundings

4 CONTINUOUS FLIGHT AUGER (CFA) PILES

Continuous flight auger (CFA) piles, which are also known as augered cast-in-place (ACIP) or augercast piles, are deep foundations in which the pile is drilled to depth in one continuous and smooth process using a continuous flight hollow-stem auger (Coleman & Arcement, 2002). The key factor in this method is to keep the auger flights filled with soil in order to maintain the stability of the hole. If the augers are turned quickly with respect to the rate of penetration into the ground, then the continuous auger acts as a sort of "Archimedes pump" and conveys soils to the surface (Brown et al., 2007). When the desired depth is reached, concrete or grout is pumped through the hollow centre of the auger pipe to the base while the augers are withdrawn from the hole.

The geotechnical conditions that favour the use of this type of foundations are generally soil profiles of medium to stiff clay soils, cemented sands or weak limestone, residual soils, medium dense to dense silty sands, and rock overlain by stiff or cemented deposits. In contrast, CFA piles are least effective in very soft soils, saturated loose or very clean uniformly graded sands, geologic formations containing voids, pockets of water, lenses of very soft soils, and/or flowing water, and hard soil or rock overlain by soft soil or loose, granular soil (Brown, et al., 2007).

5 PILE DESIGN METHODS

5.1 Introduction

The ultimate axial compression load capacity R_u of a single pile is estimated as the sum of the limit base resistance (R_b) and the frictional capacity along the pile shaft, (R_s). The static ultimate load can be computed with the following equation

$$R_u = (R_b + R_s) = \left(A_b r_b + P \sum_{z=0}^{z=L} \Delta z r_s \right) \qquad [1]$$

where A_b is pile tip area, r_b the unit tip resistance, P is the pile perimeter and r_s the unit shaft friction over a length Δz. R_u comprises the sum of the structural loads and the self weight of the pile.

5.2 Base Resistance, R_b

The limiting base resistance is conventionally calculated using bearing capacity theory or the theory of cavity expansion. In both cases, the limiting resistance is given by the equation:

$$r_b = N_b s_{ub} + \sigma_{vo} \qquad [2]$$

where N_b is a factor which varies with the theory applied, s_{ub} is the undrained shear strength and σ_{vo} is the total stress at the pile tip depth. The pile weight divided by pile area can be assumed approximately equal to σ_{vo} and so the net unit base resistance is given
by

$$r_{b,net} = N_b s_{ub} \qquad [3]$$

Considerable settlement may be required to mobilize the limit load on the pile base. It has become common practice for small diameter piles to define r_b to be the unit base resistance at a settlement equal to 10% of the base diameter. In some cases, this may be less than the true limiting base resistance. N_b values are recommended in the literature and the value of s_u to be used in equation 3 also varies depending on a number of factors including the method used to determine its value.

5.3 Shaft friction capacity, R_s

Burland (1973) suggested calculating shaft friction using an effective stress approach, commonly known as the β-method, where the unit shaft load is given by an equation of the form

$$r_s = \beta \sigma_{vo}' \qquad [4]$$

where $\beta = k_s \tan\delta$ is the shaft resistance coefficient, k_s is the final coefficient of earth pressure after pile installation and reconsolidation, δ is the angle of friction of the pile-soil interface, and σ_{vo}' the effective vertical stress at depth z_i The value of δ depends on the soil type and the properties of the pile surface. Due to the effects of pile installation, the clay adjacent to the pile is assumed to be completely remoulded, resulting in a cohesion intercept, c', of zero, and an effective angle of internal friction of ϕ'. Kulhawy et al. (1983) related values of k_s/k_o to the method of pile installation. For drilled shafts, they suggested k_s/k_o values ranging from

0.6 to 1.0, depending on the construction method and degree of workmanship.
Instead of using a unique β value for all the soil layers, Chen and Kulhawy (1994) suggested the following equation for drilled shafts:

$$\beta = \left[(1-\sin\phi')OCR^{\sin\phi'}\right]\tan\left[(\delta/\phi')\phi'\right] \qquad [5]$$

where ϕ' is determined in a triaxial compression test, OCR is the overconsolidation ratio and δ is the angle of friction between the soil and drilled shaft. They provided recommended values of δ/ϕ' that vary with the pile material. The correlation is based upon a detailed analysis of drilled shafts under axial and lateral loading.

The β-method was suggested as an improvement on the total stress method, known as the α-method, in which the unit skin friction, r_s, at any depth z_i, is related to the undrained shear strength at that depth, s_{ui}, in the form:

$$r_{si} = \alpha_i s_{ui} \qquad [6]$$

where α_i the adhesion coefficient and is a function of s_u. Published data over the last 50 years suggest that there is no unique relation between these two parameters. Recently, Coleman & Arcement (2002) have suggested the following correlation to α to be used for CFA piles installed in clay soils:

$$\alpha = 56.192 s_u^{-1.0162} \qquad (0.35 \le \alpha \le 2.5) \qquad [7]$$

5.4 Input parameters for pile design

For reliable application of the above pile design methods, the engineer has to select appropriate values of the input parameters. These will depend on soil type, the initial state and stress history of the soil, the pile geometry and material and the method of pile installation. For calculation of shaft friction, this may involve selection of k_s and δ or s_u and α. To estimate tip resistance, the engineer must select appropriate values of N_b and s_{ub}. The value of s_{ub} is typically averaged over some distance above and below the pile tip elevation. This zone of influence varies with the soil strength and stiffness and the specific design method.

As penetration testing tools such as the piezocone penetration test (CPTu) and the flat dilatometer (DMT) interact with the soil in a similar manner to piles during installation and because these in situ tests are often used to estimate the parameters used as input to the above indirect methods of pile design, several methods of pile design have evolved in which in situ test parameters such as cone penetration resistance, q_t, are adjusted using empirical factors to obtain pile base resistance and shaft friction. The empirical factors have been derived on the basis of pile load tests in a variety of soil conditions and for a variety of pile types and installation methods. These design methods are termed Direct Methods of pile design. Direct methods considered applicable to the design of CFA piles in plastic clays are examined in subsequent sections of this paper.

6 IN SITU TESTING

6.1 Introduction

The piezocone penetration test (CPTu) and the flat dilatometer test (DMT) are two of the most popular site characterization tools used in geotechnical practice. Both tests offer the advantage of a continuous profile of parameters which allow interpretation of stratigraphy as well as estimates of shear strength and stiffness. The advantages of in situ tests over conventional approaches, is that the dependency on "undisturbed" sampling and use of conventional laboratory testing is avoided (Eslami & Fellenius, 1997).

6.2 Piezocone (CPTu)

The standard piezocone has a conical tip with a 60° apex angle, is 10 cm^2 in cross-section, has a 150 cm^2 friction sleeve and pore pressure can be measured during penetration at one or more locations on or near the cone tip. The piezocone is pushed into the ground at a standard rate of 2 cm/sec and tip resistance, q_c, sleeve friction, f_s, and pore pressure, u_2, are recorded at typical intervals of 2.5 or 5 cm. The profiles of q_t, f_s and u_2 versus depth are interpreted to obtain soil stratigraphy and estimates of engineering parameters. For the work described here, a 44 mm in diameter cone with the pore pressure measured at the u_2 position was used. This is the optimum measurement location to allow correction of the measured q_c for the effects of water pressure on unequal end areas of the cone tip using the expression:

$$q_t = q_c + u_2(1-a) \qquad [8]$$

where a is the area ratio for the particular cone used.

6.3 Flat Dilatometer (DMT)

The standard DMT is a stainless steel blade having a flat, circular steel membrane mounted flush on one side. The blade is pushed into the ground using the same system as for the CPTu and with the same pushing rate. It is connected to a control unit on the ground surface by a plastic tube through which is threaded a steel wire for transmission of all electrical signals. At regular intervals (0.25 m in the work described in this paper), penetration is stopped and the membrane is expanded.

The pressures are recorded when the membrane loses contact from the sensing disc (A-reading) and when full expansion has been achieved (B-reading). The pressure when the membrane is again fully collapsed (C-reading) can also be recorded. The A, B and C readings are corrected for membrane stiffness to obtain the corrected pressures used for interpretation, denoted by p_o, p_1, and p_2, respectively. These parameters are combined to obtain the Material Index, I_D, the Horizontal Stress Index, K_D, the Dilatometer Modulus, E_D, and the Pore Pressure Index, U_D. The intermediate parameters I_D, K_D, E_D and U_D are linked to soil behaviour type and engineering parameters through correlations. For further details of the equipment and more information on the interpretation of DMT measurements the reader is referred to Marchetti et al. (2001).

7 SITE INVESTIGATION AND SOIL PARAMETERS

CPTu and DMT profiles for the test site are shown in Figures 2 and 3, respectively. Both indicated the presence of predominantly clayey soils. Within the upper 15 m, q_t and u_2 indicate the presence of sand lenses. The data also suggest that sensitive clayey silt may underlie the sand lenses. There is no evidence of the presence of the significant thickness of sands identified in the preliminary site investigation. Below about 14 m depth, under the stratified upper clay layer, a fairly homogeneous deposit of clay to silty clay extends to the maximum depth reached of 25 m.

On the CPTu profile, there is a change in consistency at about 17 m depth. The zone between about 17 and 18.5 m does not exhibit the high positive pore pressures that are present in the remainder of this layer and q_t and R_f increase. This likely indicates the presence of a desiccated zone that formed an earlier ground surface elevation. As illustrated in Figure 2 the material index parameter in the DMT profile provides confirmation of the presence of both clay layers. Even though readings with the DMT were taken at 0.25 m intervals in comparison to the 0.05 m intervals in CPTu soundings, sandy silt to silty sand lenses were also detected.

Figure 4 presents the integrated soil profile from the interpretation of in situ tests (CPTu and DMT) in combination with index parameters from laboratory tests. The K_D profile provides insight into the stress history of the soil deposit. Marchetti et al. (2001) suggested that K_D for normally consolidated clays is within the range of 1.5 to 2.0. The DMT and CPTu data suggest that the soil is overconsolidated near the ground surface. Between the crust and about 17 m, the soil is likely normally consolidated. Below 17 m, the degree of overconsolidation increases but drops again at depths below about 18-19 m, remaining in the lightly overconsolidated range to the base of the sounding. This interpretation is supported by the relationship between the moisture contents and liquid and plastic limits. The upper clay is generally of medium plasticity, and the lower layer is of high plasticity. The natural water content is considerably closer to the plastic limit in the lower clay, suggesting a greater degree of overconsolidation.

The difference between the layers is also apparent in the interpreted profiles of s_u. The undrained shear strength s_u was estimated using empirical correlations. For CPTU data, s_u was estimated from q_t using the expression:

$$s_u = \frac{(q_t - \sigma_{vo})}{N_{kt}} \qquad [9]$$

where σ_{vo} is the total vertical stress and N_{kt} is an empirical cone factor. In the absence of a site specific value of N_{kt} based on field vane or on high quality sampling and laboratory shear testing, N_{kt} was selected based on the correlation between N_{kt} and the plasticity index proposed by Aas et al. (1986). A value of N_{kt}=15 was assigned to the medium plasticity upper clay and N_{kt}=18 to the high plasticity lower clay. Also, s_u was estimated from DMT data using the empirical correlation suggested by Marchetti (1980):

$$s_u = 0.22\sigma_{vo}'(0.5K_D)^{1.25} \qquad [10]$$

The OCR profile was estimated using the empirical approaches such as the K_D-OCR correlation proposed by Marchetti et al. (2001) and the q_t-OCR relationship by Mayne (2001).

$$OCR = (0.5K_D)^{1.56} \qquad [11]$$

$$OCR = [0.305(q_t - \sigma_{vo})]/\sigma_{vo}' \qquad [12]$$

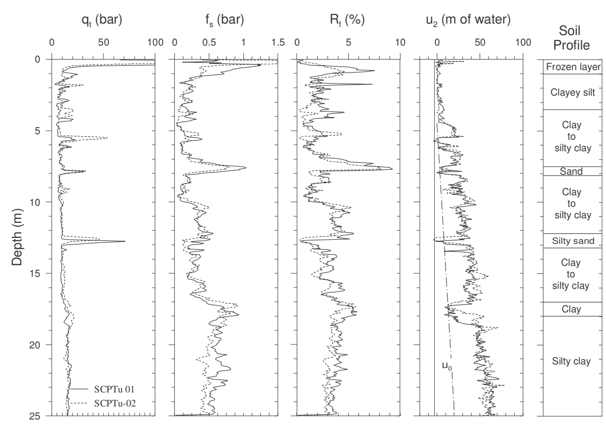

Figure 2. CPTu profiles

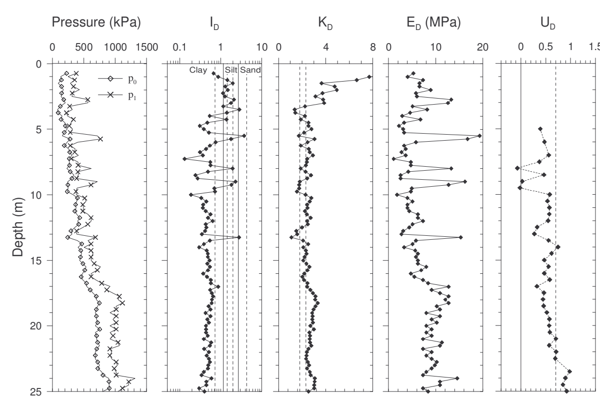

Figure 3. DMT Profile

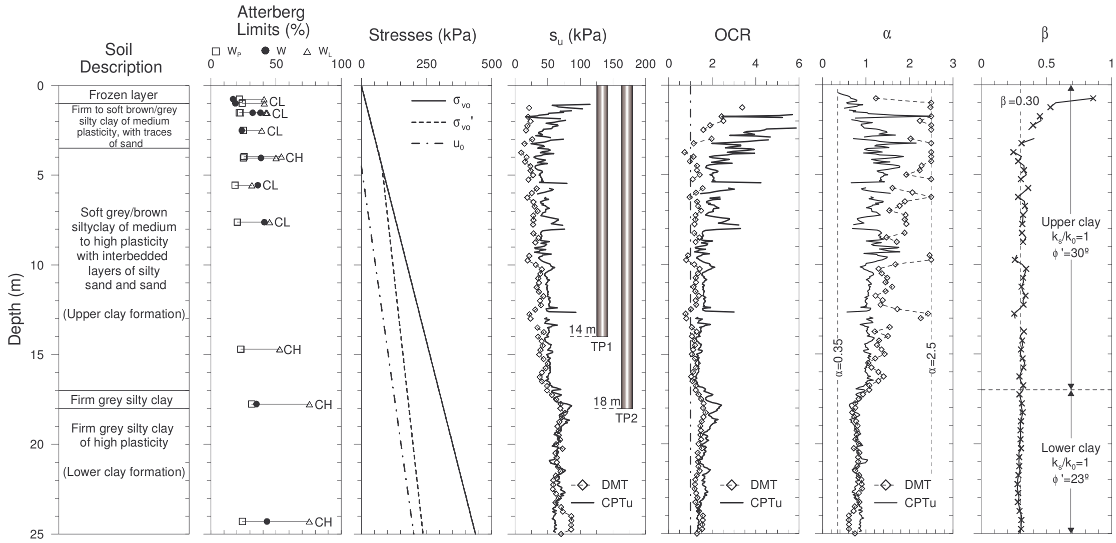

Figure 4. Integrated soil profile at the test site

8 ESTIMATES OF PILE CAPACITY

8.1 Introduction

The ultimate load capacity of each test pile was estimated using 2 indirect and 5 direct design methods. The indirect methods used were: (a) the α-method, and (b) the β-method. The following direct DMT and CPTu approaches were used: (i) DMT-C method (Powell, et al., 2001), (ii) LCPC method (Bustamante & Gianaselli, 1982), (iii) Eslami method (Eslami & Fellenius, 1997), (iv) Takesue method (Takesue et al, 1998), and (v) Togliani method (Togliani, 2008).

Most of the direct methods selected were developed using a database that included a considerable number of full scale load tests on bored piles. One exception is the DMT-C method which was developed primarily for driven or jacked steel piles in clay. Elbanna et al. (2007) previously reported good agreement between measured shaft friction from a full scale load test carried out on a bored and cast-in-place concrete pile in clay till and the values estimated using the DMT-C method. The basic characteristics of each of the direct methods used are presented in Table 1 and are discussed below.

8.2 Indirect methods

The parameters selected for use with the α and β methods are shown in Figure 4. The values of ϕ' used in Eq. [5] are based on a correlation with plasticity index presented by Terzaghi et al. (1996). Likewise, OCR values from the DMT correlation were used to estimate β values. The α values were estimated using Eq. [7].

8.3 Direct methods

8.3.1 DMT-C method

Powell et al. (2001) proposed a semi empirical design approach for the design of driven and jacked piles installed in clays. A database containing 63 pile case histories at 16 different sites in Europe was used to develop design procedures. Two direct methods based upon DMT measurements were suggested for estimation of the ultimate shaft friction capacity of piles either under compression or tension.

8.3.2 LCPC method

Bustamante & Gianaselli (1982) proposed a design procedure known as the LCPC method to estimate both end bearing and skin friction of piles from cone penetration test (CPT) measurements. The method is based upon the interpretation of experimental data from 197 full scale static loading tests. Load tests were carried out on piles built with different construction techniques (e.g. bored, driven, and grouted) and installed in a wide range of soils such as: clay, silt, sand and weathered rock. The original LCPC method was developed using values of $q_{c.}$ In the work presented here, the corrected tip resistance, q_t, is used instead as discussed by Davies (1987).

8.3.3 Eslami & Fellenius method

Eslami (1996) developed a method for estimating the ultimate pile load capacity directly from piezocone penetration test (CPTu) data. A database of 142 pile case histories around the world was used to develop the direct CPTu method. Data was collected for a wide range of pile geometries installed in different types of soil. Most of the piles were driven, but a smaller portion corresponded to concrete bored piles. In an attempt to consider the effect of pore water pressure, an "effective" cone resistance, q_E, is calculated using:

$$q_E = q_t - u_2 \quad [13]$$

A complete description of the method, and details about the selection criteria for C_t, q_{Eg} and C_s are given by Eslami & Fellenius (1997).

8.3.4 Takesue et al. method

Six pile load tests were carried out on driven and bored cast in place concrete piles at four different sites in Japan. The ultimate pile skin friction (r_s) was evaluated for all the test piles and was compared to CPTu tests carried out at each site. The soil profiles varied from silty clay to fine and medium sand. Based upon the tests results, Takesue et al. (1998) observed a direct correlation between the excess pore pressure during penetration ($\Delta u=u_2-u_o$) and r_s/f_s. The ratio, r_s/f_s, was found to be directly proportional to Δu, regardless of pile type.

The graphical relation between Δu and r_s/f_s proposed by Takesue et al. (1998) was later simplified by Mayne & Schneider (2001) to a set of analytical expressions which allow a direct estimate of r_s for both driven and bored piles. They suggested that for clays, the unit tip resistance can be evaluated using the Eslami & Fellenius (1997) method based on q_E.

8.3.5 Togliani method

More recently Togliani (2008) proposed a direct CPTu method for estimation of the ultimate pile load for driven and bored piles. The method is based upon a combination of CPT measurements, the LCPC method (Bustamante & Gianaselli, 1982) and the approach for tapered piles by Gambini (1986). Also, results from recent Pile Estimates Events (Orlando, 2002; Merville, 2003 and Porto, 2004) were used to update the proposed approach.

Table 1. Summary of equations for calculating the unit base and shaft resistance of piles with direct methods

Method	Unit tip resistance (r_s)	Unit shaft resistance (r_s)	Nomenclature
DMT-C	$r_b = k_{di} p_{1e}$ $k_{di} = 1.3 \quad E_D > 2\,\text{MPa}$ $k_{di} = 0.7 \quad E_D < 2\,\text{MPa}$	$r_s = (-1.1111 I_D + 0.775)\Delta p \quad I_D < 0.6$ $r_s = 0.11\Delta p \quad I_D > 0.6$ *For long piles where the ratio between the pile length and radius exceeds 50 (L/R>50), r_s is reduced by 15%.	k_{di}=DMT bearing factor p_{1e}=equivalent p_1 pressure beneath the base of the pile $\Delta p = p_1 - p_0$
LCPC	$r_b = q_{ta} k_c$ *for +1.5D $q_{ta}<1.3q'_{ta}$ *for -1.5D $0.7q'_{ta}<q_{ta}<1.3q'_{ta}$ *k_c and α_{LCPC} depend on the nature and stiffness of the soil as well as on the pile construction method	$r_s = q_t / \alpha_{LCPC}$	k_c=bearing capacity factor q'_{ta}= average of smoothened q_t profile 1.5 pile diameters above and below the pile tip α_{LCPC}=skin friction factor *see maximum and minimum values of k_c specified by Bustamante & Gianaselli (1982)
Eslami & Fellenius	$r_b = C_t q_{Eg}$ *see recommendations for the size of the influence zone	$r_s = C_s q_E$ *C_s is a function of the soil type and is determined from the soil profiling chart proposed by Eslami (1996)	C_t= toe coefficient C_s= shaft coefficient q_{Eg}= geometric average of the "effective" cone point resistance over the influence zone
Takesue et al.	$r_b = C_t q_{Eg}$	For $\Delta u < 300\,\text{kPa}$ $r_s = f_s\left[(\Delta_u/1250) + 0.76\right]$ For $\Delta u > 300\,\text{kPa}$ $r_s = f_s\left[(\Delta_u/200) - 0.50\right]$	C_t and q_{Eg} same as Eslami & Fellenius method $\Delta u=u_2-u_0$ u_0=in situ pore pressure f_s= CPTu sleeve friction
Togliani	$r_b = k_3 q_{t-toe}$ $k_3 = \left[0.01(L/D)\right]$ $0.3 < k_3 < 0.8$	$r_s = k_1 q_t^{0.5}$ $k_1 = 1.2\left[0.8 + (R_f/8)\right] \quad R_f < 1$ $k_1 = 1.1\left[0.4 + \ln R_f\right] \quad R_f \geq 1$ $0.75 < k_1 < 1.2$	$q_{t\text{-}toe}$=average cone tip resistance measured from 8D above and 4D below the pile tip L=length of the pile D=diameter of the pile R_f=friction ratio $R_f = (f_s/q_t) \times 100$

9 RESULTS AND DISCUSSION

Data from CPTu-01 was used to estimate the capacity of TP1 and CPTu-02 for TP2. The estimated capacities are presented in Figures 5 and 6 for TP1 and TP2, respectively. They have been normalized by the measured ultimate capacity to give the ratio, R_{up}/R_{um}. The contributions of tip resistance and shaft friction to the ultimate capacity indicate that the majority of the pile resistance is made up of shaft friction. No attempt was made to separate tip resistance and shaft friction capacity in the testing.

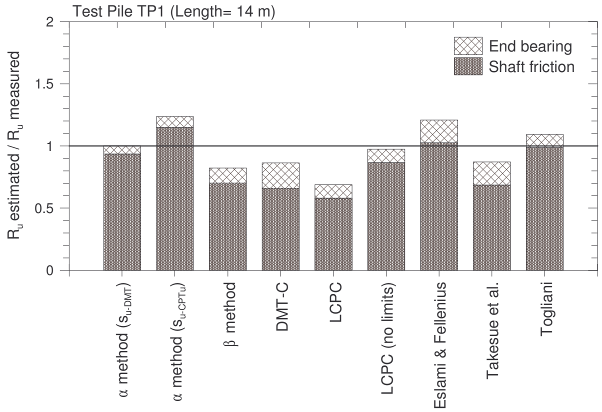

Figure 5. Estimated/Measured ultimate load capacity for the test pile TP1

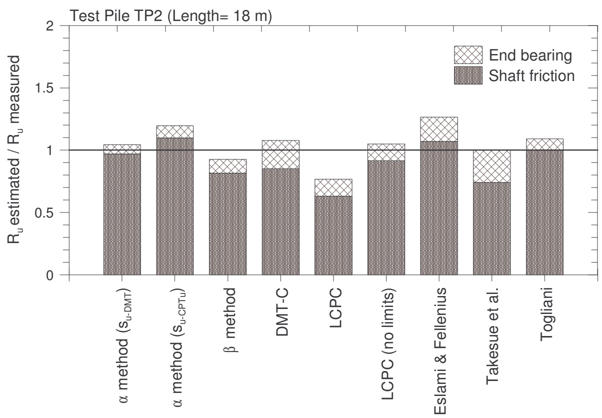

Figure 6. Estimated/Measured ultimate load capacity for the test pile TP2

Of the indirect methods, the α-method using s_u values from the DMT gave R_{up}/R_{um} values very close to 1.0 for both piles. The estimated capacity with the α-method using s_u values calculated from CPTu data gave values 17 to 24 percent larger than that from the load test. Calculations using the β-method were appropriately conservative.

A back calculated β value of 0.42 gives the best estimation of the ultimate load for the test pile TP1, whereas for the test pile TP2 the best number is obtained with β=0.37. At first sight these values seem to be quite high. However, Fellenius (2006) reported a back calculated β value of 0.5 based on results of full scale static load tests on short steel driven piles installed in soft silty clay in southern Alberta. Similarly, Elbanna et al. (2007) reported β=0.5 from back calculated results of a static load test carried out on a drilled shaft in southern Alberta clay till.

Of the direct methods, the standard LCPC approach was the most conservative. This method includes upper limits on shaft friction. When these were relaxed, the estimated capacity for the test TP1 was increased to 95 percent of the test load and the estimated load for the test pile TP2 was just 3 percent higher than the measured capacity.

For both piles, the Eslami and Fellenius method gives the highest and unconservative estimates. The other methods all produced predictions within 15% of the measured values, with the Takesue method working well for TP2 but less so for TP1.

Of the CPTu methods, the best estimations are those obtained from methods that are based upon a higher percentage of results of full scale load tests on bored piles. For example, the database of the most accurate methods: (i) Takesue (ii) LCPC, and (iii) Togliani contain data on bored piles within a range of 40 to 83 percent of the total number of tests analyzed. Furthermore, the relatively low accuracy of the Eslami and Fellenius method for CFA piles may also be attributed to the relatively low percentage, i.e. 11 percent, of bored piles case histories in the original database.

10 CONCLUSIONS

The data presented here has shown that a combination of CPTu and DMT soundings interpreted together and used in combination with results of basic laboratory tests (e.g. index tests) provide a better understanding of in situ conditions in soft soil deposits than when either in situ test is used alone.

For pile design, a reliable application of indirect design methods is dependent upon the selection of appropriate values of the input parameters by the designer. These will depend on the soil types, the initial state, stress history of the soil, and the construction method of the pile. Also, due to the variety of methods available in the literature for estimating those parameters, a subjective decision is inevitable. On the other hand, direct methods eliminate the uncertainty in the selection of those input parameters, due to the fact that direct in situ measurements are used rather than interpreted values of soil parameters.

A remarkable agreement was observed between estimates and measured values with the indirect α-method using α values calculated from the correlation by Coleman & Arcement (2002) derived specifically for CFA piles and based upon s_u values derived from the DMT correlation. However, the data presented here also proved that the parameter α is very sensitive to different values of s_u.

Of the direct methods, the LCPC method gave the closest prediction of pile capacity provided the upper limits on unit shaft friction were not applied. Application of these limits resulted in a significant underestimate of pile capacity. This suggests that restrictions may need to be adjusted or even ignored to better estimate the measured ultimate capacity at this site. Similar findings were reported by Rollins et al. (1999) for driven steel piles in clay. More recently proposed direct methods based on CPTu data show promise but require further study.

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